VOLUME 79 SEPARATE No. 326

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

NOVEMBER, 1953

FOUNDATION EXPLORATION - DENVER COLISEUM

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(Discussion open until March 1, 1954)

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Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

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This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

FOUNDATION EXPLORATION FOR THE DENVER COLISEUM

Ralph B. Peck

Introduction

The Denver Coliseum, Fig. 1, is an arch-rib reinforced concrete structure with 13 arches spaced on 28-ft centers and having a span of approximately 250 ft. It serves as an amphitheater, as a convention hall, and as an arena for the annual International Stock Show. The importance of the latter event made it essential to locate the new structure as close as possible to the existing stockyard facilities. The most suitable site was on the south side of 46th Ave. west of its intersection with 44th St. As shown in Fig. 2, the relatively flat ground in this vicinity was limited on the west by a steep bank extending downward to a pond in an old gravel pit. To facilitate the handling of traffic at the proposed Coliseum it was considered desirable to establish the structure as far to the west as possible without encroaching upon the boundaries of the old excavation.

The general appearance and character of the completed structure are shown in the photograph, Fig. 1. Each rib terminates at the top of a reinforced concrete pilaster which in turn rests upon a footing. The vertical distance from the springline of the arch to the top of the footing is 39 ft. Each footing is approximately 48 ft long, 7 ft wide and 6 ft deep. Its base rests about 7 ft below Denver City Datum or approximately 20 ft below the level of the general ground surface. Each footing is subjected to a vertical load of approximately 1400 kips, a horizontal thrust of approximately 460 kips, and a moment of about 6600 ft kips.

In addition to the usual factors of safety against sliding and bearing-capacity failure, the structural designers established certain requirements for the behavior of the foundation. Horizontal movement of any footings was not to exceed one-half inch. Rotation of any footing was not to cause a differential settlement between toe and heel greater than three-quarter inch. Differential settlement between adjacent footings was not to exceed about one-half inch.

The following discussion presents the successive steps in the program of soil exploration necessary to satisfy these requirements. A step by step procedure was utilized to provide the maximum information at the minimum expenditure of time and money.

Preliminary Data

Prior to the actual design of the structure, certain information was available that proved useful in determining the type of exploratory program to be carried out. One source of information was existing knowledge concerning the geology of the locality.

The Coliseum is located on the west slope of the valley of the present North Platte River which flows in a south-north direction through the city of Denver. Prior to the glacial epoch, the Platte River had eroded a broad and rather deep

valley in stiff clays or soft shales derived from erosion of the mountains to the west. During the glacial epoch the Platte carried vast quantities of glacial outwash and other debris from the mountains and largely filled the ancestral valley with sand and gravel. These deposits have the characteristic lenticular pattern of materials deposited by swiftly flowing streams choked with debris. This is evident at numerous exposures in the vicinity where commercial gravel pits are being or have been operated. One of these exposures is shown in the photograph, Fig. 3. In many parts of Denver the subsoil is covered by a windblown or partly windblown deposit consisting of extremely fine sand and silt with a slight amount of cohesion. Exposures in the general vicinity of the Coliseum indicated that this material should not be more than 6 to 8 ft deep.

Thus, the preliminary geological information suggested that the surficial soils should consist of fairly loose and extremely fine sands, underlain by gravel deposits likely to be erratic in their distribution and characteristics, and underlain in turn by relatively soft shales deposited prior to the glacial epoch.

A second matter of practical importance was the history of the abandoned gravel pit immediately to the west of the Coliseum site. It was known that the pit had not been operated for the extraction of gravel for more than half a century, because the remnants of a long abandoned smelter still existed within the boundaries of the pit. There were persistent rumors among old residents that slag from the smelter had been deposited at various places within and near the gravel pit, that most of this slag had subsequently been removed for road material, but that deposits of slag might still be found on the ground surface as it existed while the smelter was in operation. During the last 20 or 30 years the gravel pit had been gradually filled in. Most of the fill consisted of sand, straw and manure cleaned from stock cars after the delivery of livestock to the stockyards. The extent to which the east edge of the gravel pit had been covered by this loose and unstable material was not known, but the many years of filling suggested that much of the present ground at the proposed west end of the Coliseum site might be underlain to considerable depth by fill.

Finally, in the months prior to construction a considerable number of auger borings had been made to attempt to determine the depth of the fill. Most of these borings were very shallow and were discontinued when some obstruction was encountered. It was not generally known whether the obstructions were bricks, slag or boulders. In a few locations large pits were excavated to determine whether any extensive deposits of slag existed. Such slag as was encountered was found to be of very limited extent.

The information available prior to design is represented by Fig. 4, which shows a section along the south line of footings. Considerable uncertainty existed regarding the depth of fill beneath the entire west half of the structure, and there was at least the possibility that no firm material might be found beneath the proposed footings along the west edge. The elevation of the shale was estimated from the results of borings at a highway underpass several blocks away and was also subject to considerable uncertainty. The position of the water level was that noted at various times in the pit itself.

Figure 4 represents the type of information available at the beginning of design. Although the data contained therein were by no means adequate for design, they were extremely valuable in planning the further exploratory program to provide the maximum of information at the least expenditure.

Choice of Type of Exploratory Procedure

A study of the structural requirements together with the data in Fig. 4 indicated that whatever method of soil exploration was utilized should provide certain specific information. In the first place, the method had to be capable of determining the relative density and, hence, the bearing capacity of the natural materials. In this connection, it was important to keep in mind that the natural materials were likely to have an erratic rather than a uniform structure. In the second place, the exploratory procedure had to be such that the boundaries of the natural soil at the edge of the gravel pit would be clearly defined. This was obviously of vital importance because lack of satisfactory bearing material beneath the west end of the structure would require either a deep foundation for the end arches or would require locating the structure closer to 44th St. Neither alternative was considered desirable unless its necessity was demonstrated. Finally, the type of exploration had to be such as to locate any extensive areas of slag that might still exist.

Since it was known in advance that the fill materials would be unsatisfactory for the support of the structure and, furthermore, since it was known that the natural materials consisted essentially of sand and gravel, there was no necessity for obtaining samples of the underlying materials if their physical properties could be established in some other manner. It appeared that some type of penetration test would provide the most rapid and economical quantitative or semi-quantitative indication of the relative density of the various elements of the deposit. Yet, because of the presence of boulders in the gravel deposit up to 2 or 3 in. in diameter, with occasional larger sizes, it was believed that penetrometers of the usual small size might be unsatisfactory. Moreover, on account of the erratic nature of the deposits as well as the detailed information necessary concerning the boundary between fill and natural ground, flexibility was a requirement of the exploratory procedure. It would be desirable to explore in detail where detailed information was necessary and to make only a cursory investigation where conditions were well defined. It was especially apparent that large-diameter undisturbed sampling and testing would be out of the question on the project. The cost of even a few large-diameter samples of the sand and gravel would be prohibitive. Furthermore, because of the erratic nature of the deposit, it seemed more important to determine the pattern of the relative density rather than to obtain precise values of the physical constants for a few samples that would almost certainly not be representative of the deposit as a whole.

It was decided that to satisfy these requirements further exploration should be carried out by means of a large-diameter penetrometer. To avoid as much interference from the boulders as possible, the base diameter of the penetration cone was made 2-1/2 in. To deliver sufficient energy to obtain penetration even if the gravel were dense, the driving hammer was assigned a weight of 350 lbs and the height of fall was set tentatively at 2 ft. The penetrometer was to be driven into the ground at the bottom of two-inch OD drill rods. In this manner a clearance of about one-quarter inch would normally be left around the rods. This would minimize the effect of friction due to the material surrounding the rods, and would facilitate withdrawal of the rods after completion of a penetration test. The penetrometer point itself was expendable and would remain in the ground after each record was obtained. It was the intention to drive the penetrometer continuously from the ground surface until the shale was reached at a depth of about 50 ft, to count the number of blows of the driving hammer required for each foot of penetration, to withdraw

the drill rods and to move to another location. By eliminating the operation of drilling a hole or of taking a sample, and by abandoning the cone at the bottom of each hole, it was believed that the operation could be carried out with such speed as to make it relatively inexpensive. The details of the penetrometer are shown in Fig. 5.

Exploratory Program

Arrangements were made to start the penetration tests along the south line of footings at the east end of the structure. The east end was chosen because it was farthest from the gravel pit and it was believed that some time should be permitted to develop experience with the use of the penetration device before the complex conditions near the gravel pit were approached. Indeed, as shown in Fig. 6, the first two soundings did not penetrate deeper than about 35 ft. However, the field crew soon became familar with the technique, and subsequent soundings were completed with little difficulty. Two soundings were made at the location of every second footing from east to west. Thus, pairs of soundings were spaced at 56 ft in the east-west direction. The results of the soundings for the south line of footings are shown in Fig. 6, including the results of two soundings at footing 15, 28 ft from the proposed west end of the building. These two soundings were made after the others were completed to assist in defining the uncertain conditions at the west end.

It became apparent, as the information contained in Fig. 6 was developed, that the gravel deposit encountered at about El. -7 or -8 was extremely dense and that it had a thickness of approximately 10 ft. Below that level the penetration resistance decreased to about 30 or 40 blows per ft until at about El.-36 a very high resistance was encountered. This indicated the shale. It was concluded that the material in the upper 5 or 6 ft of the easternmost soundings was fill, that the first resistant material represented the original ground surface, and that the underlying material was the loose fine sand which rested directly upon the dense gravel. Thus, in the eastern part of the building, the presence of a dense gravel layer was established at about El. -8 where it was originally desired to establish the footings.

On the other hand, although at footing 16 the gravel layer existed, at footing 14 there was no indication of such a layer. Indeed, to about El. -22 the material had extremely low penetration resistance and was undoubtedly the sand and manure fill. The soundings at footing 15 suggested that the gravel had been removed to about El. -12 and that above this level was fill.

When this information became available, a detailed exploration of conditions in the vicinity of the west end of the structure was started. During this time daily liaison was maintained by telephone between the field crew and the foundation engineers. In this manner the conditions could be precisely defined without getting superfluous information.

This part of the program may be demonstrated by means of Fig. 7, on which the penetration records for footings 14, 15 and 16 are copied from Fig. 6. As the first step, a pair of soundings was made 10 ft west of footing 15. These indicated no gravel to about El. -18. Two more sounding were made 5 ft to the east of footing 15. In one of these it appeared that gravel was encountered at about El. -12, and in the other at about El. -16. Finally, a pair of soundings was made 10 ft east of footing 15. These disclosed gravel at the normal elevation. Thus, conditions were quite clearly established. It was now apparent that footings at the normal elevation could not safely be established on the gravel if their left-hand edge was closer than about 38 ft from

the center line of the proposed footing 14. Similar studies for the north side of the structure led to similar conclusions and indicated that the entire structure would have to be moved 50 ft to the east to provide a footing foundation located uniformly on the upper surface of the extremely dense part of the gravel deposit. In the face of this evidence, the location of the structure was changed.

Figure 1 shows the locations of the soundings described in the preceding paragraph and also all the other soundings made at the site of the structure. No resistance indicative of slag was found in any of the soundings, and the rumors of large areas underlain by slag were discredited.

Concurrently with the soundings three standard penetration tests were made with a split spoon sampler in order to check the classification of the materials against that assumed on the basis of the local geology, and in order to get, if possible, a correlation between the number of blows in the special penetrometer and that in the standard penetration test. The locations of the three standard penetration test borings are shown in Fig. 1. The results are shown in Fig. 8. Each of the diagrams in Fig. 8 shows the results of the individual values obtained from the standard penetration tests together with the results of the closest continuous soundings made with the special device. It is seen that the correlation is exceptionally good. It was not anticipated that the number of blows in the special tests would correspond numerically to that in the standard penetration tests, but rather it was anticipated that there might be a factor by which one value should be multiplied to obtain the other. However, merely by chance, the factor relating the two penetration resistances was found to be unity. This made it possible to use the results of the penetration tests without modification for determining the allowable soil pressures.

Basis for Selecting Soil Pressure

In the interpretation of the standard penetration test the significant value of the penetration resistance in blows per foot is the minimum average within a depth below footing level approximately equal to the width of a footing. At the Coliseum site, this minimum value occurred at footing 22 and was equal to 34. For all other footings supported on sand or gravel, higher values of penetration resistance were indicated. Various correlations indicate that the angle of internal friction for a penetration resistance of 34 blows is on the order of 37°. Considerations of ultimate bearing capacity (see, for example, Theoretical Soil Mechanics by K. Terzaghi, page 125) lead to the conclusion that the ultimate capacity of footings having a width of 7 ft, and located at a depth of at least 10 ft below the surrounding surcharge level, would be on the order of 30 tons per sq ft. Therefore, there was no danger of the footings breaking into the ground, and considerations of settlement governed the design. An estimate of the settlement was made in accordance with Fig. 177, Soil Mechanics in Engineering Practice, by K. Terzaghi and Ralph B. Peck. According to this figure, a footing 7 ft wide, subjected to a uniform soil pressure, would experience a settlement of about 1 in. at a pressure of 7400 lbs per sq ft. The actual pressure beneath the Coliseum footings varies from a minimum at the heel to a maximum of about 6360 lbs per sq ft at the toe. From these values, it may be concluded that the settlement of the toe of the footing should hardly exceed 3/4 in. and that the differential settlement between toe and heel should be less than this amount. This differential settlement would correspond to a rotation of the base of the footing on the order of 0.0012 radians. Since the pilasters are located near the heels of the footings

it is also apparent that the differential settlement between adjacent footings near the pilasters should not exceed 1/2 in.

Observed Movements

No specific measurements were made to determine the differential settlements or the total settlements of the footings themselves. However, these movements may be inferred from observations of the rotation of the pilasters combined with the outward movements of the tops of the pilasters.

The inclination of approximately vertical gage lines on the three westernmost pilasters on each side of the structure was established shortly after construction of the pilasters and before the concrete arch sections were cast. The inclination was observed again after the arch centering had been struck and the full dead load became active. The top of the pilasters moved out with respect to the bottom such as to produce an average rotation amounting to 3 minutes with a maximum rotation of 5 minutes. A rotation of the pilaster of 5 minutes corresponds to a rotation of the footing of .00145 radians, on the assumption that both the pilaster and the footing remain perfectly rigid. This value may be compared with the anticipated maximum of 0.0012 radians. Actually, the flexibility of the structure would involve a smaller differential settlement between toe and heel than indicated by the measurements. At the time of decentering the maximum horizontal translation at the top of a pilaster was 0.25 inches. The combination of this movement with the rotation of the pilaster results in practically no horizontal movement of the footings.

Conclusion

The development of the penetration tests at the site of the Denver Coliseum afforded a rapid, inexpensive, and practical means for determining the essential characteristics and boundaries of an erratic granular deposit. In spite of the high penetration resistance of the gravel layer itself, it was possible to make from two to three soundings per day to a depth of 50 ft. The borings for the standard penetration tests, on the other hand, required about three days each. It is believed that the procedures adopted at the site were well suited to the local conditions and that, for the given expenditure of time and money, more definite and reliable information was obtained than would have been possible for a conventional program of sampling and testing.

Acknowledgment

Roberts and Schaefer Company of Chicago and A. Gordon Lorimer of New York were Associated Engineers and Architects for the structure. Roland L. Linder of Denver was Consulting Architect. Mr. Richard Bennetts was Coordinator of Construction, City and County of Denver. The soundings were made by the Raymond Concrete Pile Company. Mr. Henry Oetzman, foreman.

To be presented at June meeting, ASCE, Denver, 1952.

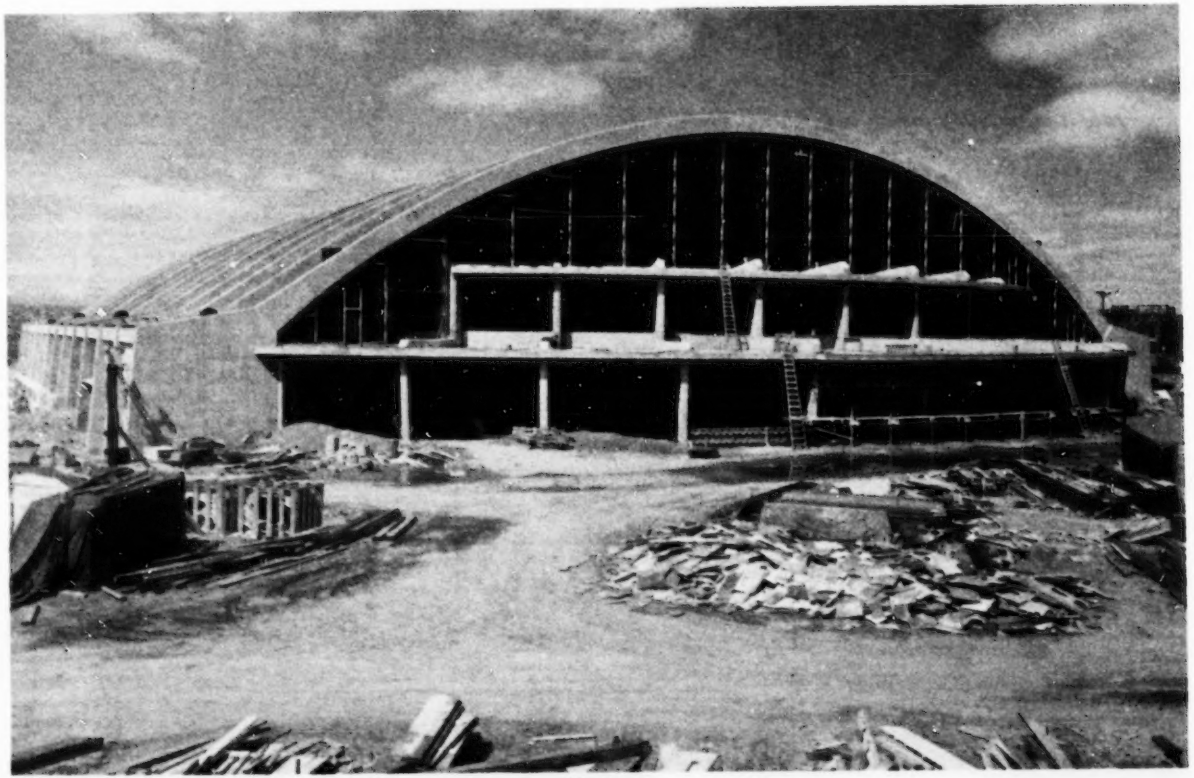


FIG. 1

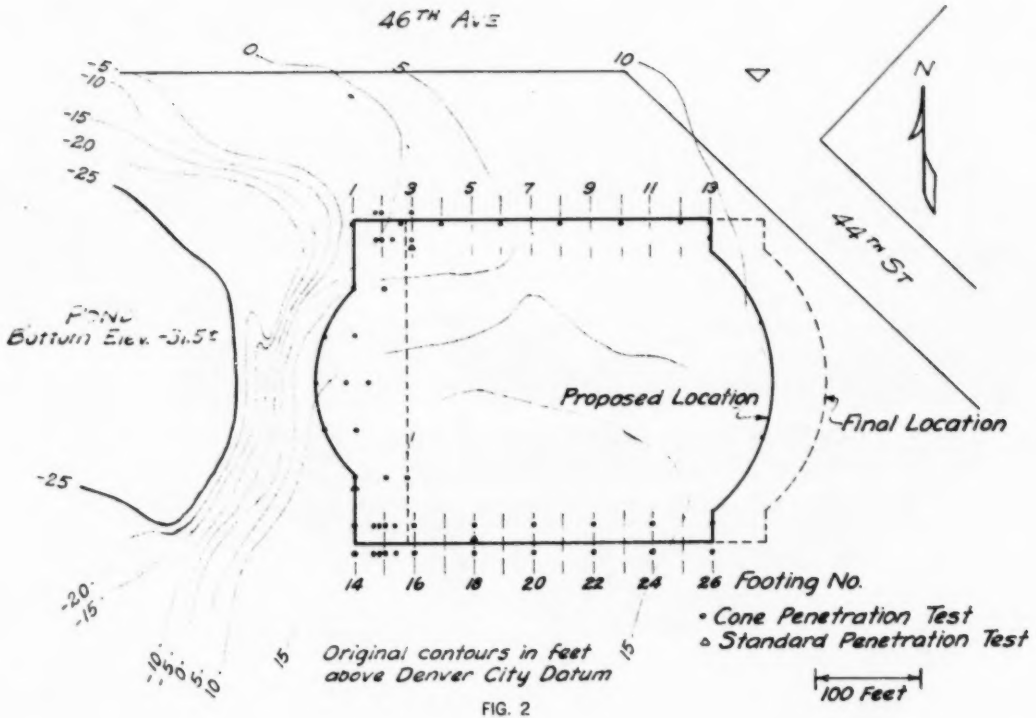

46TH AVE
44TH ST
N
0
5
10
-5
-10
-15
-20
-25
1
3
5
7
9
11
13
14
16
18
20
22
24
26 Footing No.
Proposed Location
Final Location
Bottom Elev. -31.5
• Cone Penetration Test
△ Standard Penetration Test
Original contours in feet above Denver City Datum
100 Feet

FIG. 2

FIG. 3

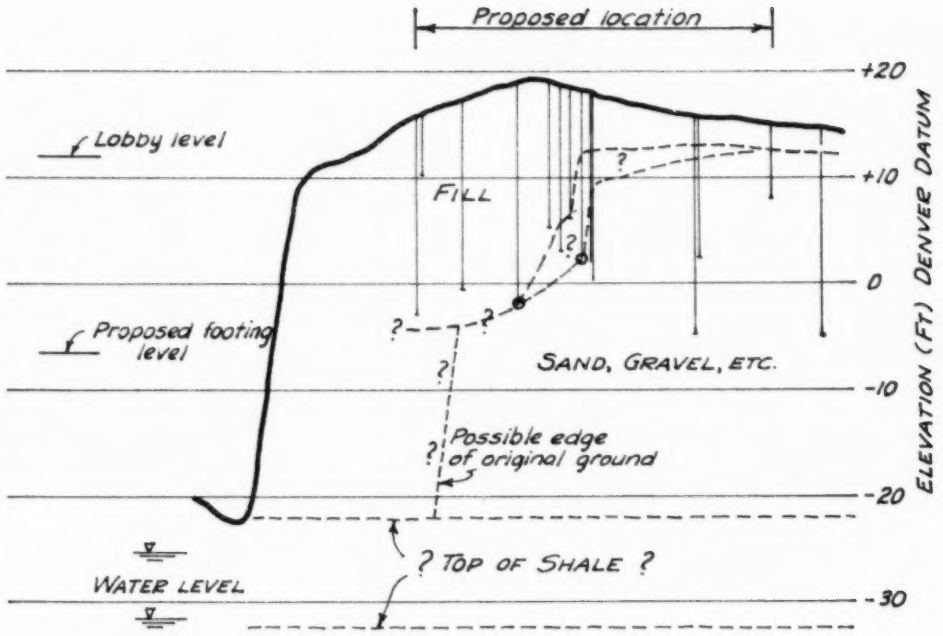

Proposed location
+20
+10
0
-10
-20
-30
ELEVATION (Ft) DENVER DATUM
Lobby level
Proposed footing level
FILL
SAND, GRAVEL, ETC.
Possible edge of original ground
? TOP OF SHALE ?
WATER LEVEL
Vertical lines indicate auger borings
⊙ indicates open pit
100 Feet

FIG. 4

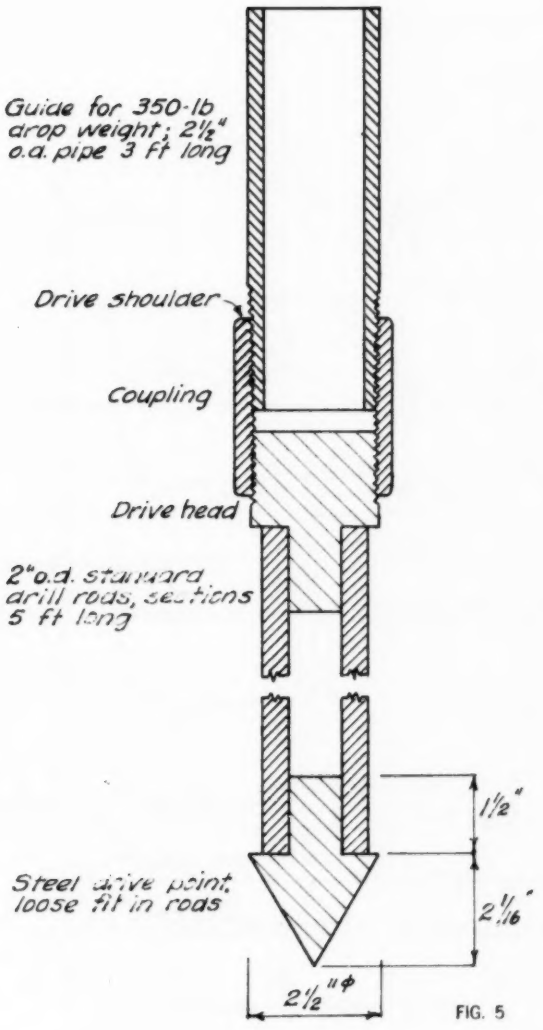
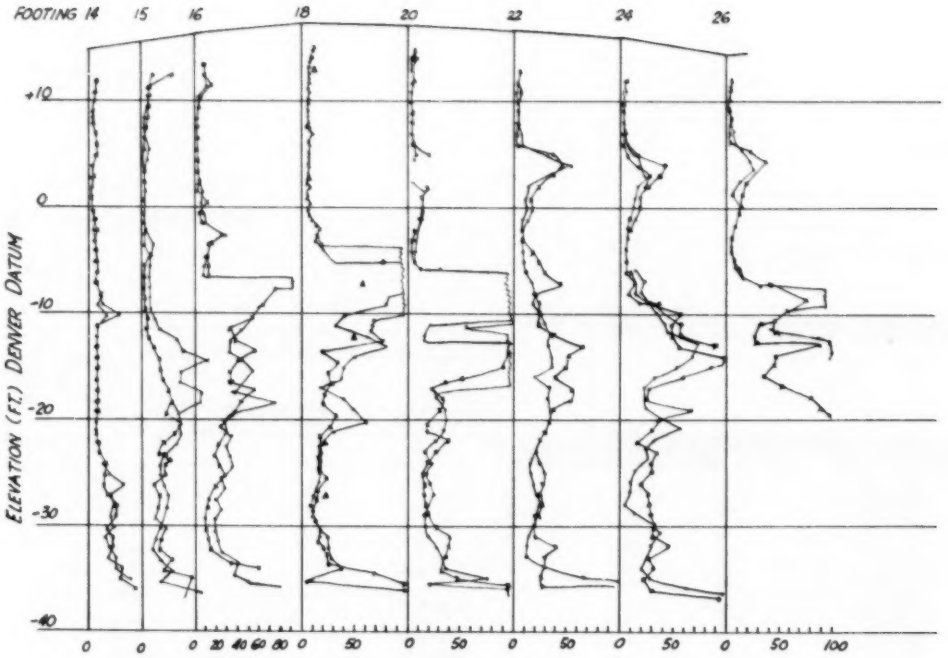

Guide for 350-lb drop weight; 2½" o.d. pipe 3 ft long
Drive shoulder
Coupling
Drive head
2"o.d. standard drill rods, sections 5 ft long
Steel drive point, loose fit in rods
1½"
2 1/16"
2½" ϕ

FIG. 5



- South
- North
- North (2nd Attempt)
- Gow Boring

Horizontal Scale : 1" = 28'-0" = 1 Bay

FIG. 6

DENVER STADIUM
RESISTANCE DIAGRAMS
SOUTH FOOTINGS

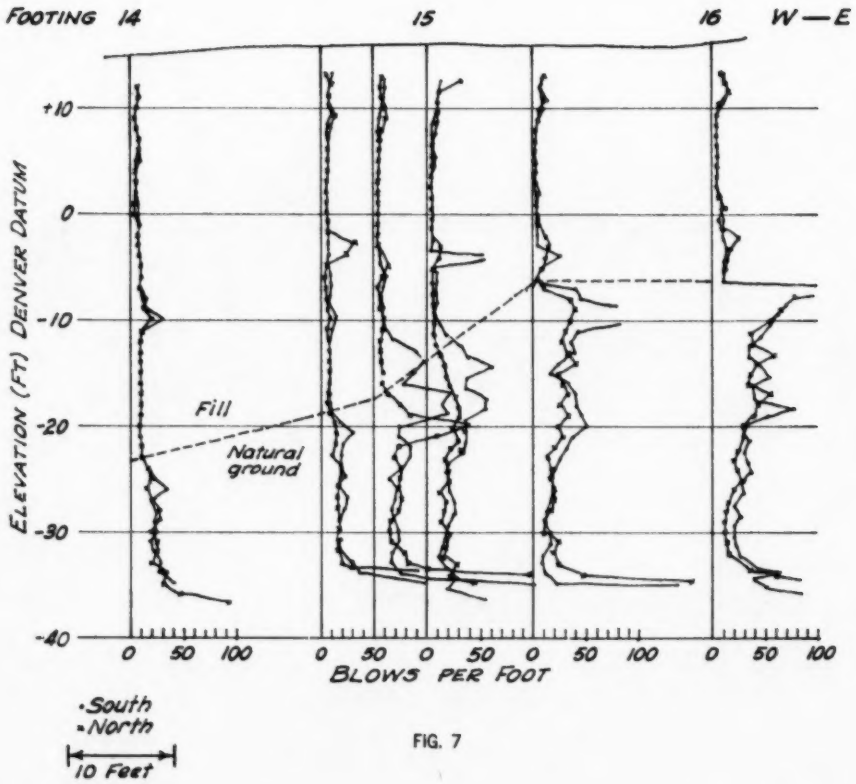
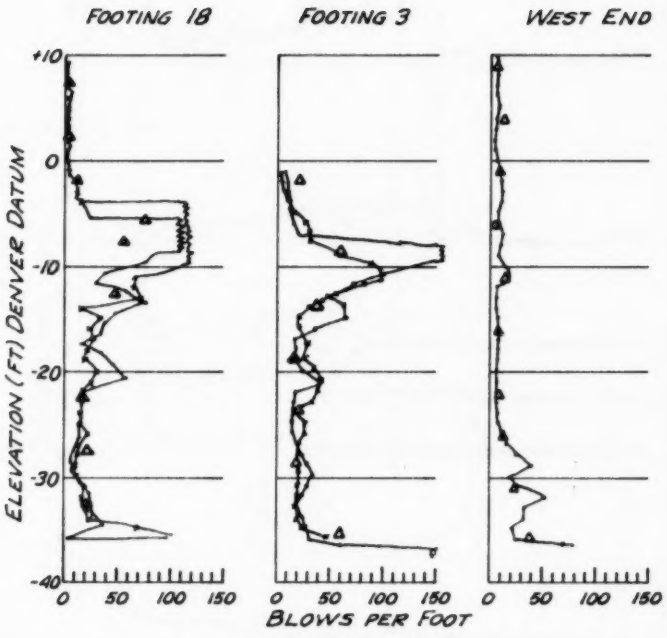

FOOTING 14
15
16
W — E
+10
0
-10
-20
-30
-40
ELEVATION (FT) DENVER DATUM
Fill
Natural ground
0 50 100
0 50
0 50
0 50 100
0 50 100
BLOWS PER FOOT
•South
▪North
10 Feet

FIG. 7



△ Standard Penetration Test Value

FIG. 8